LABORATORY MODEL TESTING ON SUCTION PILES IN CLAY FOR MOORING OF MOBILE OFFSHORE BASES

S. Bang, T. Preber, J. Thomason, and Y. Cho
Department of Civil and Environmental Engineering
South Dakota School of Mines and Technology
Rapid City, SD, USA

ABSTRACT As part of the feasibility study of constructing Mobile Offshore Bases, suction piles are currently being studied to provide the necessary mooring capability. This paper presents the preliminary results of the experimental laboratory model tests on suction piles in clay. The results have been used to calibrate the mobilized soil adhesion included in the analytical simulation of the suction pile installation, i.e., suction pressure vs. pile penetration relationship. The mobilized soil adhesion is described as a function of a non-dimensional parameter.

En partie d'une étude de possibilité de la construction des fondements mobiles au large, on étudie actuellement des pilotis aspirants pour pouvoir la capabilité nécessaire pour s'amarrer. Cette étude présente les résultats préliminaires des épreuves modèles laboratoires expérimentales sur des pilotis aspirants dans du sol argileux. On a employé les résultats pour calibrer l'adhésion du terrain mobilisé compris dans la simulation analytique de l'installation du pilotis aspirants, c'est-à-dire le rapport de la pression d'aspiration contre la pénétration des pilotis. L'adhésion du terrain mobilisé est décrit comme une fonction d'un paramètre nondimesionnel.

1. INTRODUCTION

The US Office of Naval Research is currently conducting a feasibility study program to advance critical design technologies for Mobile Offshore Bases (MOB's). The MOB is expected to be a self-propelled, floating, propositioned base that could sustain for long-term deployment. The dimensions of the MOB are approximately 1,500 meters by 120 meters with the internal storage space of 800,000 m^2.

The South Dakota School of Mines and Technology participates in the MOB feasibility study to provide an adequate mooring technique for this very large floating structure. Since the vertical and lateral loads expected from the MOB is to be extremely large in magnitude, any conventional underwater mooring technique may not provide adequate resistance. For this reason, suction piles that have been introduced recently are currently being investigated to identify whether they can provide the necessary mooring capability.

Suction piles typically have a large diameter (up to 32 meters have been used to date) with a relatively small length to diameter ratio. It is installed by applying a suction pressure inside the pile, which results in a net external surcharge that pushes the pile into the seafloor. The details of the suction pile with regard to its use, mechanism, installation, and analysis and design methods can be found in references (Burgess et al. 1981, Burgess and Hird 1983, Hogervost 1980, Morrison and Clukey 1994, Senpere and Auvergne 1982, Tjelta et al. 1986).

This paper describes the details of the laboratory experimental model tests on suction piles in a clayey seafloor. The test facility, test details, results, and calibration of the analytical solution method are included.

2. TEST FACILITY

2.1 Soil Preparation

The soil used was kaolinite at high moisture content and low shear strength in order to simulate the ocean seafloor. The clay was prepared by consolidating slurry for different periods of time using sealed containers with internally applied vacuum.

The clay slurry was mixed from EPK (a division of Feldspar) brand porcelain grade kaolinite mixed with water to produce a material with 120 to 130 percent water content by weight. Mixing was done by pouring kaolinite and water into a 50-gallon barrel, and circulating the slurry from the bottom of the barrel through a rotary pump and back into the top of the barrel. Mixing was done until the slurry achieved a creamy texture with no evidence of clots.

Following completion of the mixing, the slurry was poured into a container made of 305 mm flex pipe (flexible air duct pipe without insulation) affixed to a 30.5 cm brass sieve pan (Figure 1). The pan was filled with sand and a filter fabric was fitted on top of the sand. A hole was drilled in the pan wall below the fabric and a valve stem was attached to provide vacuum. The top of the flex tube was fitted with a 20 mm thick plywood disk and the edge between the disk and the pipe was sealed with silicone. The flex pipe was placed within a rack in order to provide a uniform vertical consolidation.

Following completion of the assembly, 500 to 600 mm Hg of vacuum was applied to the bottom of the sample through the valve stem. Consolidation time varied between 10 and 20 days depending on the shear strength distribution desired. Records of sample height versus time were kept throughout the entire consolidation process.

2.2 Model Pile and Setup

The model pile consisted of a 50 mm diameter and 300 mm long Plexiglas pipe with a wall thickness of 6 mm. The tip of the pile was beveled at an angle of approximately 30 degrees with the longitudinal axis of the pile. The top of the pile was capped with a Plexiglas disk, and a vacuum line leading to a vacuum pump was attached through the center of the plate. A pressure transducer to record the level of vacuum was attached near the top of the pile and on the pipe outside the pile. The vacuum line was equipped with two moisture collection chambers and a dessiccator chamber to minimize the amount of moisture entering the vacuum pump. A vacuum control valve was attached to the line between the dessiccator chamber and the vacuum pump. The experimental setup is shown schematically in Figure 2.

Penetration of the pile was measured either manually or with the use of an LVDT attached to the top of the pile.

3. EXPERIMENT DETAILS

3.1 Pile Installation

The pile was set up over the flex pipe following removal of the plywood disk and then manually pushed into the clay to a preset initial penetration depth. A guide was used to keep the pile in a vertical position. Vacuum was then applied and gradually increased until penetration started.

3.2 Test Procedure

The pile was tested using a 119.5 gram or a 250 gram surcharge to compensate for the low self weight of the pile. In addition, initial penetrations of 37.5 and 75 mm were used.

After the pile was seated, all instruments were set to zero readings on the digital data acquisition system (DAS). A small amount of vacuum (5 to 20 mm Hg) was applied to the pile to initiate movement. After the pile stopped penetrating, a four minute pause was allowed to assure that all significant movement ceased, and the corresponding movement and vacuum were recorded. Vacuum was then carefully increased in increments by opening and adjusting the vacuum control valve (Figure 2) until the pile started to move again. The vacuum level was then maintained at that level until movement ceased. As long as the pile moved, manual pressure readings were taken at every 3.2 mm. DAS increments were set at 10 second intervals. Pile movement generally occurred in increments of 3 to 15 mm or less.

4. ANALYTICAL SOLUTION

To successfully penetrate the suction pile into the seafloor, the soil resistance must be overcome. The resistance of the pile is the pile bearing capacity corresponding to the state of the pile penetration. The resulting pile penetration depth at any given applied suction pressure can therefore be determined from the equilibrium. Equilibrium requires that the bearing capacity of the pile equal to the external force including the weight of the pile, applied surcharge, and suction pressure. When a constant suction pressure with the resulting total external force exceeding the pile bearing capacity is applied, the pile starts to penetrate until it reaches a depth where the pile bearing capacity equals the external force. As the suction pressure increases again, the external load also increases and the pile starts to penetrate into the soil until the next equilibrium is reached. This procedure repeats until the pile installation is completed or the pile does not penetrate any further. It is noted that during the installation process the suction pressure should not exceed the critical pressure that may induce the instability of the clay inside the pile, i.e., plugging which occurs when the clay column separates at the base of the pile and therefore renders the suction pressure ineffective.

Clay near the pile may be disturbed due to the continuous pile penetration. This will result in a reduction of the clay cohesion/adhesion between the soil and the pile. To quantify this reduction in clay cohesion/adhesion, the concept of the "mobilized soil cohesion ratio", β, has been introduced. It is defined as

$$\beta = \frac{c_m}{c_u} \qquad [1]$$

where

c_m = mobilized soil cohesion necessary for the equilibrium between the external force and the pile bearing capacity, and
c_u = soil undrained shear strength.

The variation of β has been determined from the results of laboratory tests by matching the calculated pile penetration with the observed pile penetration at given conditions.

4.1 Pile Bearing Capacity

The pile bearing capacity can be determined from the pile tip bearing capacity and the frictional capacity. Depending upon the pile diameter to length ratio, the soil inside the pile may behave as a unit with the pile or independent to the pile. The total bearing capacity of the former case will be the sum of the tip bearing capacity based on the gross cross-sectional area of the tip and the frictional capacity developed outside the pile minus the buoyant weight of the soil inside the pile. The latter case however should consider the tip bearing capacity based on the net cross-sectional area of the tip and the frictional capacity developed both inside and outside the pile. The total pile bearing capacity, Q, therefore can be expressed as the smaller of these two cases, i.e.,

$$Q = \text{minimum}\,[Q_1, Q_2] - W_{pile} \qquad [2]$$

where

$Q_1 = Q_{outside} + Q_{inside} + Q_{net,\ tip}$
$Q_2 = Q_{outside} + Q_{gross,\ tip} - W_{insde\ soil}$
$Q_{outside}$ = frictional capacity between the outside surface of the pile and the soil,
Q_{inside} = frictional capacity between the inside surface of the pile and the soil,
$Q_{net,tip}$ = tip bearing capacity of the net cross-sectional area of the pile,
$Q_{gross,tip}$ = tip bearing capacity of the gross cross-sectional area of the pile,
$W_{insde\ soil}$ = effective weight of the soil plug inside the pile,
W_{pile} = effective weight of the pile.

4.2 End Bearing Capacity

For an undrained condition, the general ultimate bearing capacity, q_u, is expressed as

$$q_u = c_m N_c + q N_q \qquad [3]$$

where

q = overburden at the tip of the pile, and
N_c and N_q = bearing capacity factors

The general ultimate bearing capacity equation is further modified as shown below to include the effects of the pile shape and the depth.

$$q_u = c_m N_c F_{cs} F_{cd} + q N_q F_{qs} F_{qd} \qquad [4]$$

4.3 Bearing Capacity Factors

The bearing capacity factor N_c proposed by Prandtl (1921) and N_q by Reissner (1924) are shown below.

$$N_q = 1 \qquad [5]$$
$$N_c = 5.14 \qquad [6]$$

4.4 Shape and Depth Factors

The shape factors can be evaluated by the equations suggested by De Beer (1970) as shown below.

$$F_{cs} = 1 + .2\frac{B}{L} \qquad [7]$$
$$F_{qs} = 1 \qquad [8]$$

The depth factors extended by Hansen (1970) used in this study are shown below.

$$F_{cd} = 1 + 0.4\ k \qquad [9]$$

$$F_{qd} = 1 \qquad [10]$$

where

$$k = \frac{D_p}{D} \qquad \text{for} \quad \frac{D_p}{D} \le 1$$
$$k = \tan^{-1}\left(\frac{D_p}{D}\right) \qquad \text{for} \quad \frac{D_p}{D} \ge 1$$

D_p = pile penetration depth, and
D = pile diameter

4.5 Pile Frictional Capacity

The pile frictional capacity, q_s, can be estimated from

$$q_s = \sum_{i=1}^{N} \int_0^{L_i} \pi D f_s dz \qquad [11])$$

where

D = pile diameter,
$f_s = c_m$ or $c_u \cdot \beta$,
L_i = pile embedment length within the soil layer I, and
N = number of soil layers

5. EXPERIMENTAL RESULTS

Figure 3 shows the direct relationship between the mobilized soil cohesion ratio (β) and the applied suction pressure for all model tests conducted. Figure 4 describes the relationship between β and the pile penetration depth. The effect of the suction pressure can be seen from these two figures, i.e., the value of β generally decreases as the suction pressure or the pile penetration depth increases.

As should be expected, since the undrained shear strength varies both from test to test and with depth, there is a considerable data scatter. Therefore, a better description for the variation of β needs to be made. For this purpose, a non-dimensional term is introduced. It includes the effects of the pile dimensions, the soil cohesion at the pile tip, and the applied equivalent external weight due to the suction pressure and the surcharge. It is expressed as

$$NP = \frac{(P_s + F_b / A)D}{c_u D_p} \qquad [12]$$

where

P_s = applied suction pressure inside the pile,

F_b = equivalent external weight,
A = net pile cross-sectional area,
D = pile diameter,
c_u = undrained clay strength at the pile tip, and
D_p = the pile penetration depth.

As can be seen from Figure 5., the mobilized soil cohesion ratio can be expressed very well with little data scatter when it is plotted against the non-dimensional term.

Figure 6. shows the variation of β versus undrainded shear strength c_u. As can be seen, β decreases with increasing shear strength as expected. However, the adhesion is also a function of the relative displacement between the pile and the soil. Thus, as the pile penetration increases, the clay next to the upper portion of the pile become more disturbed, hence the resistance between the pile and the soil would decrease.

6. CONCLUSIONS

Experimental laboratory model tests on suction piles have been conducted to provide calibration necessary for establishing an analytical solution between the suction pile penetration in a clayey seafloor and the applied suction pressure inside the pile. Details on the experiment setup, test procedures, test results, and the calibration of the mobilized soil internal friction angle are described.

Experiments indicate that suction is very effective in penetrating piles in clay. However, the applied suction is limited due to the possible soil instability within the pile, i.e., plugging. It was also observed that the state of soil next to the pile became disturbed due to the pile penetration. Therefore, the conventional bearing capacity equations may not be used directly. Mobilized soil cohesion ratio is therefore introduced to describe the average reduction in soil undrained shear strength.

An analytical solution is also formulated to establish the relationship between the pile penetration and the applied suction pressure, using the mobilized soil cohesion, which in turn is described as a function of the mobilized cohesion ratio. The mobilized cohesion ratio is expressed as a function of the non-dimensional term, which includes the effects of the pile dimensions, the soil cohesion at the pile tip, and the applied external load. The experimental results are used to calibrate the mobilized cohesion ratio.

The calculated values of the mobilized soil cohesion are found to be very reasonable. However, additional experiments must be conducted in order to identify additional factors that may influence the value of the mobilized soil cohesion, such as the pile diameter to length ratio. A 75 mm diameter model pile is currently being tested with various initial penetration depths, surcharge weights, and soil undrained shear strength.

7. ACKNOWLEDGMENTS

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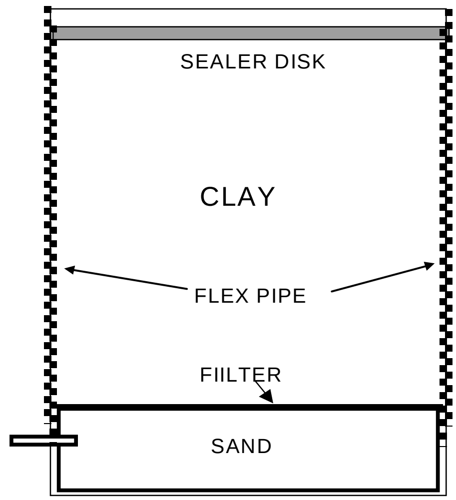

Figure 1. Consolidation Assembly

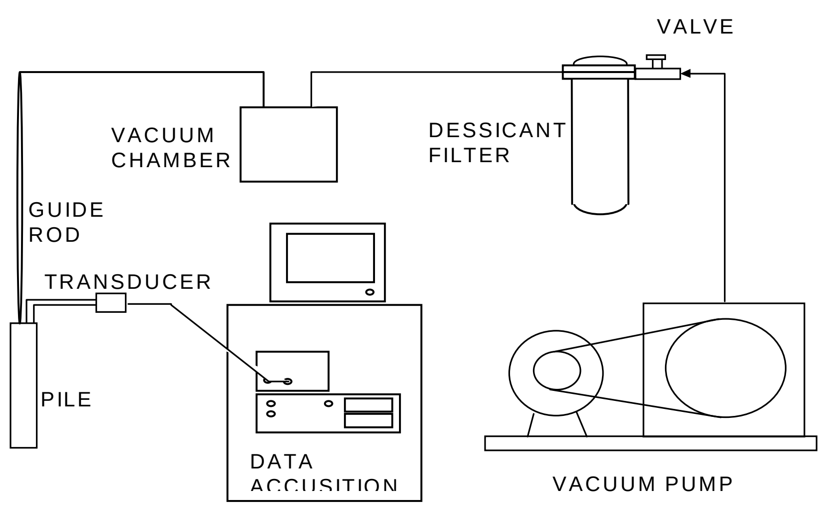

Figure 2. Schematic of Experimental Setup

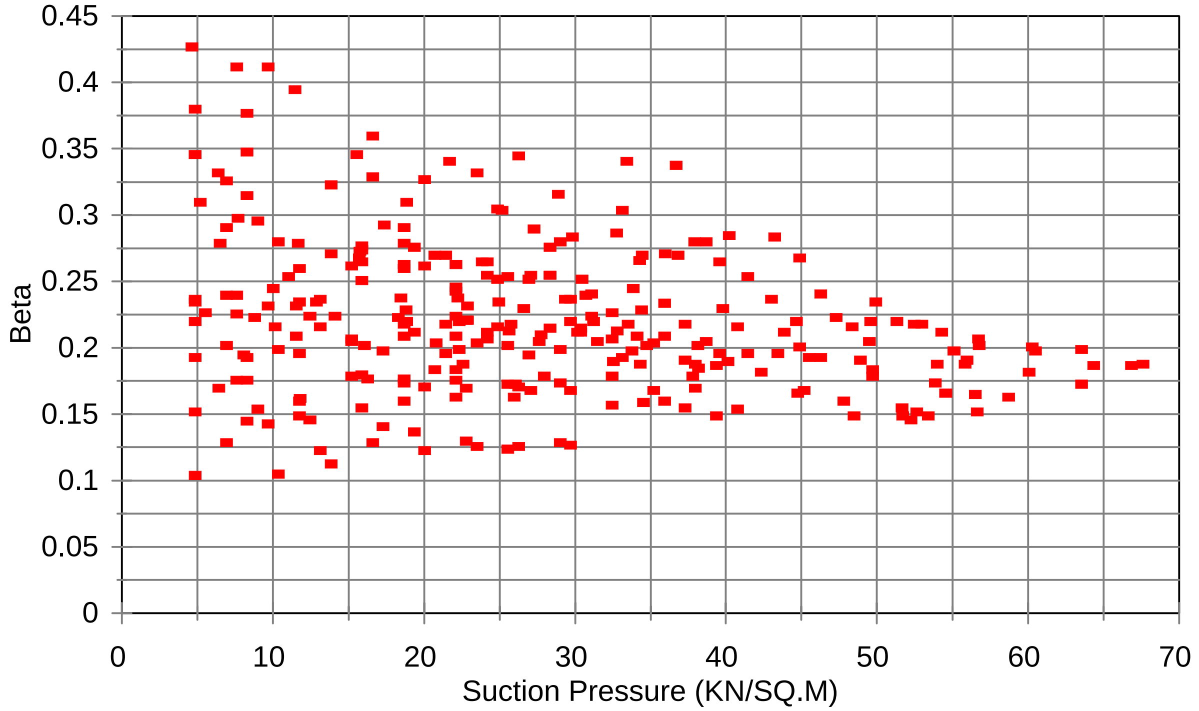

Figure 3. Beta versus Applied Suction Pressure

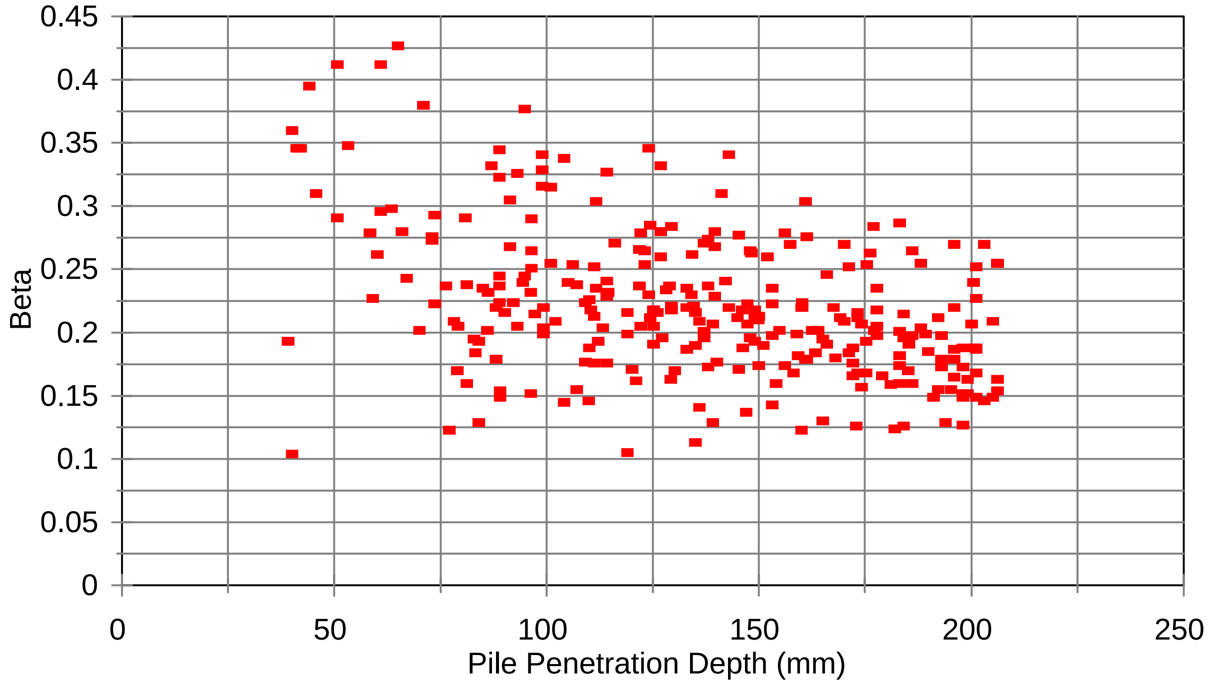

Figure 4. Beta verus Pile Penetration Depth

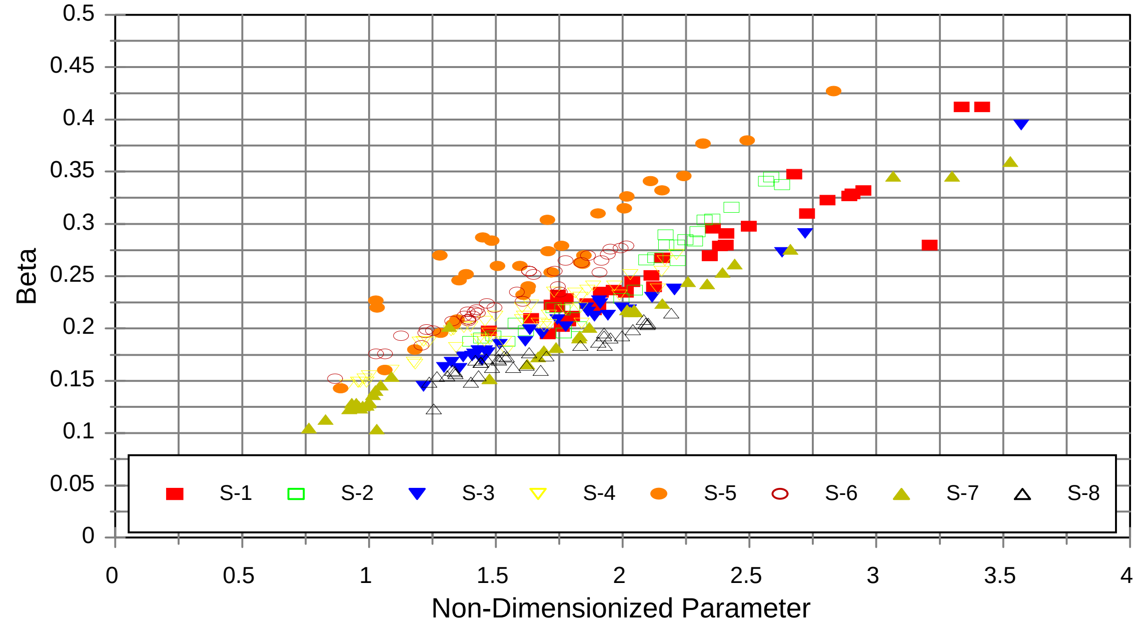

Figure 5. Beta versus Non-Dimensionalized Parameter

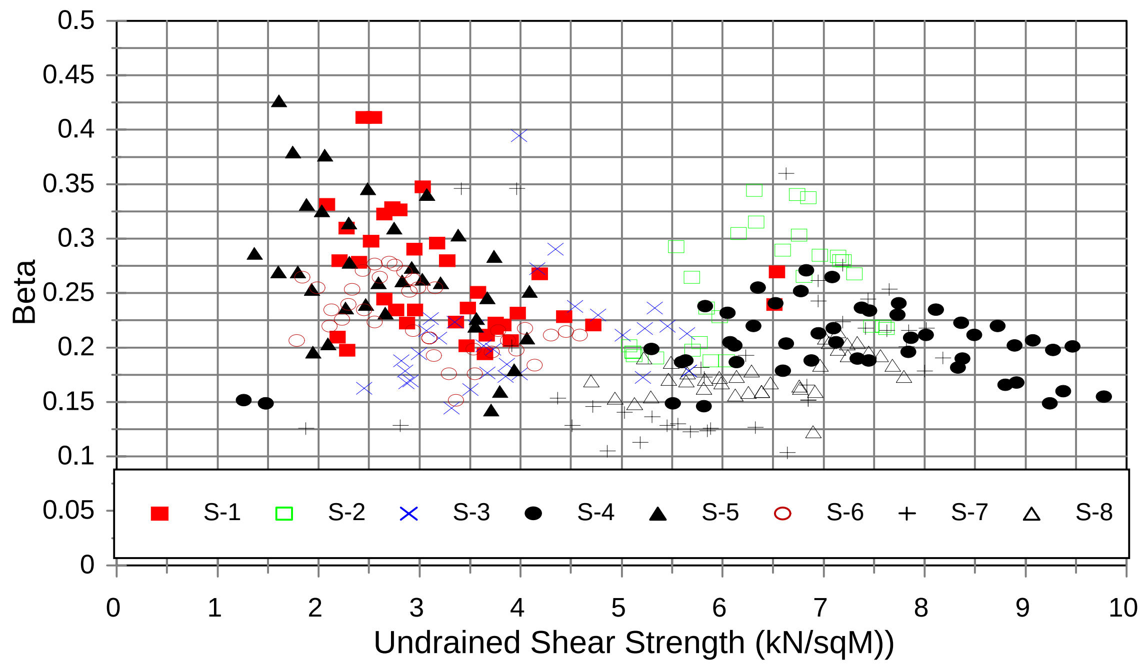

Figure 6. Beta versus Undrained Shear Strength